BEHAVIOR OF STRIP FOOTING ON MULTI LAYERED GEOGRID REINFORCED SAND BED

A PROJECT REPORT SUBMITTED IN PARTIAL FULFILLMENT
OF THE REQUIREMENTS FOR THE DEGREE OF

Bachelor of Technology
in
Civil Engineering

By

MANAS MOHANTY

Department of Civil Engineering
National Institute of Technology
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Under the Guidance of

Dr.C.R. Patra

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CERTIFICATE

This is to certify that the thesis entitled, "Behavior of strip footing on multi-layered geogrid reinforced sand bed" submitted by Sri Manas Mohanty in partial fulfillment of the requirements for the award of Bachelor of Technology Degree in Civil Engineering at the National Institute of Technology, Rourkela (Deemed University) is an authentic work carried out by him under my supervision and guidance.

To the best of my knowledge, the matter embodied in the thesis has not been submitted to any other University / Institute for the award of any Degree or Diploma.

Date

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May, 2007

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ABSTRACT

Soil Reinforcement is an effective and reliable technique for improving the strength and stability of soil. The reinforced soil or mechanically stabilized earth is a compacted soil fill, strengthened by the inclusion of tensile elements like geogrids, geotextiles, metal bars and strips. It is now well established in heavy construction industry for the construction of structures like retaining walls, embankments over soft soil, steep slopes etc.

Several papers relating to the evaluation of the ultimate and allowable bearing capacities of shallow foundation supported by geogrid reinforced sand and saturated clay have been published. This thesis pertains to the study of the behavior of centrally loaded strip foundation on multi layered geogrid reinforced sand bed.

Laboratory model test results for the ultimate bearing capacity of a strip foundation supported on multi-layered geogrid reinforced sand and subjected to central loading are presented. Only one type of geogrid Tensar BX1100 and one variety of sand at one relative density were used. The ultimate bearing capacities obtained from model tests have been compared with the theory given by Huang and Menq (1997)

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Chapter 1

INTRODUCTION

The reinforced soil is the soil in which the metallic, synthetic or geogrids are provided to improve its engineering behavior. The technique of ground improvement by providing reinforcement was also in practice in olden days. Babylonians built ziggurats more than three thousand years ago using the principle of soil reinforcement. A part of the Great Wall of China is also an example of reinforced soil construction. Basic principles underlying reinforced soil construction was not completely investigated till Henry Vidal of France who demonstrated its wide application and developed a rational design procedure. A further modified version of soil reinforcement was conceived by Lee who suggested a set of design parameters for soil reinforced structures in 1973.

Binquet and Lee (1975) investigated the mechanism of using reinforced earth slab to improve the bearing capacity of granular soils. They tested model strip footings on sand reinforced foundations with wide strips cut from household aluminum foil. An analytical method for estimating the increased bearing capacity based in the tests was also presented. Fragaszy and Lawton also used aluminum reinforcing strips and model strip foundations to study the effects of sand and length of reinforcing strips on bearing capacity.

In this thesis, the results of experimental studies on cohesion less soil, reinforced with geogrids has been presented. Tests have been conducted with the provision of geogrids in different layers up to four layers at various spacing and the results have been compared with the results of unreinforced condition. The experimental values have been compared with the theoretical values obtained from Huang and Menq equation, 1997.

Chapter 2

LITERATURE REVIEW

2.1 REVIEW OF LITERATURE

Guido et al. (1980) performed a series of laboratory model tests on rectangular and square footing. They indicated that bearing capacity ratio (BCR) at a settlement of 0.1B increases rapidly with increasing strip length up to a length of about 0.7B after which it remains relatively constant. They also found similar conclusions using square sheets of geogrid to reinforced sand.

Omar et al. (1993) have conducted laboratory model test results for the ultimate bearing capacity of strip and square foundations on sand reinforced with geogrid layers. Based on the model test results, the critical depth of reinforcement and the dimensions of the geogrid layers for mobilizing the maximum bearing capacity ratio have been determined and compared. From this experiment, they have drawn conclusions that for development of maximum bearing capacity, the effective depth of reinforcement are 2B for strip footings and 1.4B for square footings. Further they have observed that maximum width of reinforcement layers for optimum mobilization of maximum bearing capacity ratio is 8B for strip footings and 4.5B for square footings.

Dash et al. (2001) have presented the laboratory test results of strip footings on geocell reinforced sand beds with additional planar reinforcement. The test results show that a layer of planar geogrid placed at the base of the geocell mattress further enhances the performance of the footings in terms of the load carrying capacity and the stability against rotation. The beneficial effect of this planar reinforcement layer becomes negligible at large heights of geocell mattress. From the experiments they have drawn conclusions that the cumulative beneficial effect of geocell mattress and planar geogrid layer is found to be maximum for $h/B = 2$, where h = depth of reinforcement from the base of footing and B = width of footing. The overall performance improvement reduces with the reduction in the base friction and interlocking of the encapsulated soil in the geocell pockets with the sub-grade soil through the aperture openings of basal geogrid.
Mandal and Manjunath (1994) have conducted an extensive program of monotonically

loaded footings. The study is aimed at investigating the effects of a single layer of geosynthetics reinforcing material on the improvement of bearing capacity and settlement characteristics of strip footings under plane strain conditions supported by compacted sand and also to study the effectiveness of placing the reinforcing layer horizontally and vertically. The bearing capacity increase due to the use of a geosynthetic layer has been expressed in terms of a non dimensional bearing capacity ratio (BCR). The study shows that the BCR could be improved up to 1.8 times when reinforcement is suitably located relative to the footing. The horizontal reinforcement is found to be more effective in improving the bearing capacity as compared to the vertical reinforcement.

Shin et al. (2001) have done laboratory test to determine the bearing capacity of strip footing supported by sand reinforced with multiple layers of geogrid of one type. The results show that the ratio of the critical depth of reinforcement below the footing w.r.t the width of footing is about 2. For a given reinforcements depth ratio, the BCR w.r.t ultimate load increases with the embedment ratio of the foundation.

Dash et al. (2001) presented the results from laboratory model tests on a strip footing supported by sand reinforced with a geocell mattress. The parameters varied in the testing program include pattern of geocell formation, pocket size, height and width of geocell mattress, depth of the top of geocell mattress, tensile stiffness of the geogrids used to fabricate geocell and the relative density of sand. With the provisions of geocell reinforcement, failure is not observed even at a settlement equal to 50% of the footing width and a load as high as 8 times the ultimate bearing capacity of the unreinforced sand. The performance improvement is significant up to a geocell height equal to 2 times the width of the footing. Beyond that height, the improvement is marginal. The optimum width of the geocell layer is around 4 times the footing width at which stage the geocell would intercept all the potential rupture planes formed in the foundation soil.

Busehehrian and Hataf (2003) have performed tests to investigate the bearing capacity of circular and ring footings on reinforced sand along with numerical analysis. The effects of the depth of the first layer of reinforcement, vertical spacing and number of reinforcement layers on bearing capacity of the footings are investigated. Both the experimental and numerical studies have indicated that when a single layer of reinforcement layer is used, there is an optimum reinforcement embedment depth for which the bearing capacity is greatest. There is also an optimum vertical spacing of vertical layers for multilayer reinforced sand. The bearing capacity also increases with increasing number of reinforcement layers, if they are placed within a range of effective depths. They have drawn the conclusions that for circular footings on reinforced sand, the maximum bearing capacity occurs at different values of u/D and z/D depending on the number of reinforcement layers and are not unique as reported by some researchers, where u = depth of first layer of reinforcement, z = vertical distance between layers, D = circular footing diameter or outer diameter of ring footing. For ring footings, however it has been concluded that increasing the number of reinforcement layers leads to the decrease of the optimum values of z. Also it is concluded that a maximum threshold exists for the effect of the rigidity of reinforcement and therefore choosing a more rigid reinforcement does not always lead to better results in terms of BCR for circular and ring footings on reinforced sand.

Dash et al. (1994) have done model studies on circular footing supported on geocell reinforced sand underlain by soft clay. The test beds are subjected to monotonic loading by a rigid circular footing. The influence of width and height of geocell mattress as well as that of a planar geogrid layer at the base of the geocell mattress on the overall performance of the system has been systematically studied through a series of tests. The test results indicate that the provision of geocell reinforcement in the overlaying sand layer improves the load carrying capacity and reduces the surface heaving of the foundation bed substantially. The performance improvement increases with increase in the width of the geocell layer up to b/D = 5, beyond which it is negligible. (Here, b = width of geocell layer, D = diameter of footing). The overall performance improvement is significant up to a geocell height of about two times the diameter of the footing and beyond this, the improvement is marginal.

Benrabah et al. (1996) have done analytical work and undertaken experiments to explore the changes in stress distribution for a sand medium reinforced by geomembrane layers. The three results i.e. 1.from experiments; 2.using the Boussinesq method and 3.using the fast lagrangian analysis of continua calculation program (FLAC) are established. They have found that even in the case of loaded reinforced soil, the vertical stress distribution follows Boussinesq's formula, especially for high loads and despite the discrete nature of the experimental medium being used. This may be due to the fact that in the foundation zone, the soil grains are structured in such a way that they closely approximate a continuous medium. In the case of loaded reinforced soil, the presence of reinforcing layers has no bearing on the vertical stress, while the horizontal stress shows a large increase. The use of FLAC program leads to slightly lower calculated stress values, however the increase of the horizontal stress can be predicted by FLAC.

Aigen Zhao (1996) has presented the failure criterion for a reinforced soil composite. The failure criterion of reinforced soil presented here is anisotropic due to inclusion of geosynthetic reinforcement with preferred direction. The slip line method in relation with the derived failure criteria can be used for calculating the failure loads of geosynthetic reinforced soil structures. The stress characteristics of reinforced slopes, retaining walls and foundations are presented and compared with those without reinforcement. The inclusion of geosynthetic reinforcement enlarges the plastic failure region in a reinforced soil structure and significantly increases the load bearing capacity.

Palmeira et al. (1998) have stated that geosynthetics reinforcement can be used to increase the factor of safety of embankments on soft soils, particularly for shallow foundation layers. They have presented back analysis of some reinforced embankments that can be found in literature using stability methods commonly employed. The results obtained suggest that these simple methods are useful tools for predicting factor of safety of reinforced embankments when the required input data are available and accurate.

Lopes and Ladeira (1996) have studied the interaction between well graded gravelly sand and a uniaxial geogrid. The influence of the confinement pressure, soil density and displacement rate on the pull out resistance of the geogrid is discussed by analyzing the results of the pull out tests. The major conclusions of this analysis:-

1. An increase in the confinement stress, soil density or displacement rate increases he pull out resistance of the geogrid.
2. The increase in the pull out resistance when the confinement stress increases, is not in proportion to the increase in the normal stress because there is reduction in the adherence factor.

Chunsik yoo (2001) has presented the results of laboratory model tests on the bearing capacity behavior of a strip footing supported by a geogrid reinforced earth slope. A wide range of boundary conditions including unreinforced conditions, are tested by varying parameters such as geogrid length, number of geogrid layers, vertical spacing and depth of top most layer of geogrid. He results are then analyzed to establish both qualitative and quantitative relationships between the bearing capacity and the geogrid parameters. He has found that for reinforced slope loaded with a footing, the failure zone tends to become wider and deeper than that for the unreinforced slope. The bearing capacity of a footing situated on the crest of sloping ground can be significantly increased by the inclusion of the layers of geogrid as in the level ground.

Zhao et al. (1997) have shown the utility of geogrid reinforcement in stabilizing the weak subgrade of a liquid retention pond in Nebraska. The results of the field tests indicates that the multilayer geogrid is capable of providing the subgrade support needed to achieve the required placement and compaction of a 0.6m thick lagoon clay liner over a weak subgrade. The test also confirms that without the geogrid, the required compaction cannot be achieved. The use of multi layer geogrid in this project has eliminated a 0.3m crushed stone layer which resulted in a significant savings in material and excavation cost.

Manjunath and Diwakar (1994) have conducted experimental investigation to determine the influence on the performance of a strip footing located at and near the crest of a granular slope fill with the inclusion of reinforcing layer within the body of the fill. The investigations have been carried out by varying edge distances of the footing for three different slope angles and for two types of geogrids. Both load settlement behavior and the ultimate bearing capacity of the footing show considerable improvement by the inclusion of a reinforcing layer at appropriate location in the body of the sloped fill. The conclusion drawn from the tests are:

1. The insertion of a geogrid reinforcement layer at suitable location of the sloped fill will considerably increase the load carrying capacity as well as stability of footing on slopes.
2. At an edge distance of 5.0B, the ultimate bearing capacity of footing becomes independent of the slopes.
3. At any given slope angle, the footing on reinforced slope exhibits much higher load bearing capacity than that of unreinforced slope.

Andrzej sawicki (1998) has proposed a simple model of the elasto-plastic soil reinforced with the visco-elastic geosynthetics. Two modes of reinforced soil behavior are considered within the framework of mechanics of composite materials. The first mode corresponds to the elastic soil (E-V mode) and the second one to plastic soil (P-V mode). During the E-V mode, the initial stress in reinforcement decreases, causing the regrouping of micro stresses in the soil down to the stage when a yield condition in the soil is reached. Then the P-V mode begins during which the stress in reinforcement remains constant but the macro strains increase due to creep. Here a method of estimating the initial stresses in rheological soil has been presented.

Gurung (2003) has done a laboratory study on the ensile response of unbound granular base road pavement model using geosynthetics, this paper explains the structural response of a pavement base layer under applied tensile forces in a laboratory set up. Experimental results show higher tensile resistance of geosynthetics included in the pavement base and such high tensile responses will significantly influence the behavior of the pavement surfaces. The tensile peak strength of a pavement reinforced with geogrid is higher than pavement reinforced with geotextile. The high tensile strength of a reinforced pavement will have significant increase on the behavior of the pavement surfaces under traffic loading and environmental movements.

Raymond and Ismail (2003) have shown that, the improvement of bearing capacity of track, highway and runaway embankments on unbound aggregate can be done using geogrid. Geogrid reinforcement in unbound aggregate will improve the performance of the transportation support. They have presented experimental results for three different construction possibilities of geogrid reinforcement in the unbound aggregate layers. The aggregate layers are subjected to both repeated loading and static loading. In a uniform deposit of granular aggregate statically loaded by a surface footing, with the introduction of a single layer of reinforced material, the beneficial effect on load bearing capacity of the reinforcement decreases from a depth of 0.0625 times the footing width and is negligible at a depth of about one half of the footing width.

Alawaji (2000) has presented the settlement and bearing capacity of geogrid reinforced sand over collapsible soil. The potential benefits of geogrid reinforced sand over collapsible soil, to control wetting induced settlement was investigated. Model load tests have been carried out using a circular plate of 100mm diameter and geogrids. The width and depth of the geogrids have been varied to determine their effects on the collapse settlements deformation modulus and bearing capacity ratios. The results show that there is significant difference in structural contribution of the tested geogrid which ranges from 95% reduction in settlement, 200% increase in elastic modulus and 320% increase in bearing capacity. It is found that efficiency of sand geogrid system is increased with increasing geogrid width and decreasing geogrid depth with respect to bottom face of the footing up to a certain range, after that there is no such improvement. For efficient and economical reinforcement of sand pad over collapsible soil, geogrid of width 4D and depth 0.1D are recommended. (Here, D = diameter of loaded area)

Das et al. (1998) have conducted laboratory tests to find out the effect of transient loading over a foundation supported by geogrid reinforced sand. In the test, a square foundation is used and through out the test one relative density is maintained. In al the tests, the peak value of the transient load per unit area of the foundation exceeded the ultimate static bearing capacity of foundation supported by unreinforced sand. The conclusion drawn this test is that the geogrid reinforcement reduces the settlement due to transient loading.

Patra et al. (2005) have done laboratory tests to determine the ultimate bearing capacity of embedded strip foundation on geogrid reinforced sand. The results have been compared with the bearing capacity theory developed by Huang and Menq (1997). Based in the tests, the following conclusions were drawn:

1. For the same soil, geogrid and configuration, the ultimate bearing capacity increases with increase in embedment ratio, D_f/B.
2. The theoretical relationship for ultimate bearing capacity developed by Huang and Menq (1997) provides somewhat conservative predictions.

It is recommended that tests of this type be carried out for karstic soils and weak cohesive soils to evaluate the improvement in bearing capacity, which may be helpful in field conditions.

Kumar and Saran (2003) have conducted laboratory tests on closely spaced strip and square footings on geogrid reinforced sand. The study was carried out to evaluate the effect of spacing between the footings, size of reinforcement and continuous and discontinuous reinforcement layers on bearing capacity and tilt of closely spaced footings. The interference effects on bearing capacity and settlement of closely spaced square footings on geogrid reinforced sand were almost insignificant in comparison to those on isolated footings on reinforced sand, whereas, a significant increase in the tilt of adjacent square footings has been observed by providing continuous reinforcement layers in the foundation soil under the closely spaced square footings. A considerable increase in bearing capacity, settlement and tilt of adjacent strip footings has been observed by providing continuous reinforcement layers in the foundation.

Shin et al. (2002) have conducted small scale laboratory model tests to determine the ultimate bearing capacity of a strip foundation supported by sand with multiple layers of geogrid reinforcement. The embedment ratio of the foundation was varied from zero to 0.6. It is found that, for the given reinforcement-depth ratio, the bearing capacity ratio w.r.t ultimate load increases with embedment. The bearing capacity at limited levels of settlement is smaller than the value at ultimate load.

The following conclusions were drawn:-

1. The critical reinforcement-depth ratio below the bottom of the foundation $(d/B)_{cr}$ for deriving the maximum benefit from reinforcement is about 2.
2. For a given reinforcement-depth ratio, u/B, h/B and b/B, the baering capacity ratio w.r.t to the ultimate load (BCR_u) increases with the embedment ratio of the foundation (D_f/B).

Jun Otani et al. (1998) have analyzed the bearing capacity of geosynthetics reinforced cohesive foundation soil reinforced soil loaded by a flexible strip footing using a rigid plastic finite element formulation. This method is based on the upper bound theorem of the theory of plasticity and bearing capacity is obtained as a load factor at the ultimate limit state. The reinforcing material and the surrounding sand are modeled as a single composite material with an equivalent cohesion. The underlying soft ground is assumed to be purely cohesive and hence both the reinforced soil and soft ground are modeled using the Von-Mises failure criterion. The method of analysis used here is firstly verified for the case of unreinforced ground by comparison with the Prandtl solution. For the case of reinforced cohesive foundation ground, a series of analysis are conducted by changing the depth and length of the embedded geosynthetics as well as its number of layers. Based on these layers, the reinforcing effect on the bearing capacity of the cohesive foundation grounds is evaluated. From the analysis, it is concluded that the bearing capacity of the geosynthetic reinforced foundation ground is increased as the depth and length of the reinforcement are increased, but there is an optimum depth at which the maximum reinforcement effect is obtained. There is also an optimum number of geosynthetics layer. The results obtained here are quantitatively checked against the field measurements and a design chart also developed for estimation of bearing capacity of geosynthetic reinforced foundations on soft ground.

Shin et al. (2001) conducted field tests on land reclaimed from the ocean for the construction of Incheon International Airport in Korea. The field test arrangement was a plate load test on a granular pad with and without Geogrid reinforcement. The magnitude of stress is measured below the granular pad and the boundary surface of stress, σ distribution is inclined at an angle α with the vertical. They have observed that for all values of Q/A, the magnitude of α increases substantially, when reinforcements are in place. The reinforcement layers help redistribute the stress, σ over a larger area and reduce its intensity. This, in turn, allows the foundation to support a larger load per unit area for a given settlement level.

Patra et al. (2006) published the results of a limited number of studies for the ultimate bearing capacity of strip foundation supported by Geogrid-reinforced sand and subjected to eccentric loading. They varied the eccentricity ratio (e/B) from zero to 0.15 along with the foundation embedment ratio (D_f/B) from zero to 1.

The following conclusions were drawn:-

1. For similar reinforcement conditions, the ratio of the ultimate bearing capacity of eccentrically loaded foundations to that loaded centrally can be related by a reduction factor.
2. The reduction factor is a function of D_f/B and e/B.

2.2 GEOGRIDS

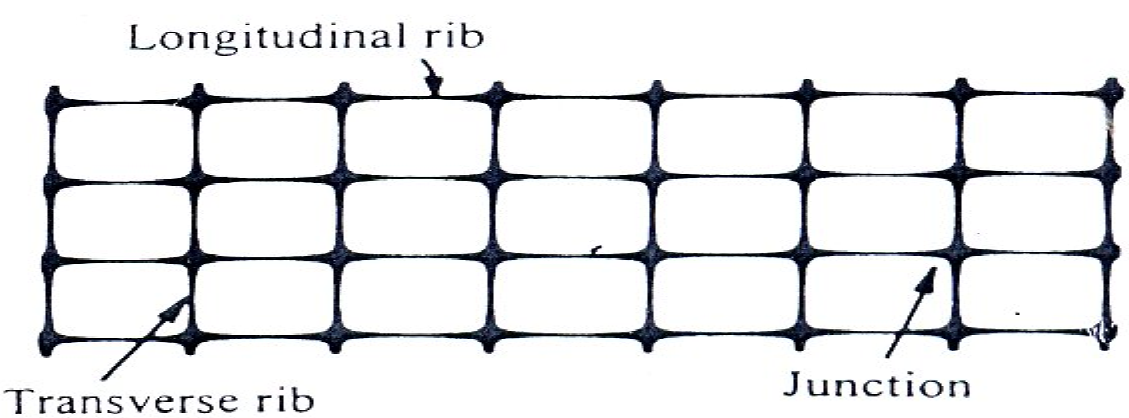
1. Geogrids are relatively stiff net like material with large openings called apertures.
2. The apertures are large enough to allow interlocking with the surrounding soil and rock to perform the function of reinforcement.
3. They are incorporated in the base layers of paved or finished surfaces, or in surface layers of walls and slopes and provide a stabilizing force within the soil structure itself.
4. This stabilization occurs as the fill interlocks with the grid. The interlocking effect is determined by the geogrid strength; mesh size and base materials used.
5. Geogrids are made of high modulus polymer materials like polypropylene (PP) and high density polyethylene (HDPE) and are prepared by tensile drawing.

2.2.1 Types of Geogrids

1. Biaxial Geogrid

- Manufactured by stretching the punched sheet of polypropylene in 2 orthogonal directions.
- Has high tensile strength and modulus in 2 perpendicular directions.

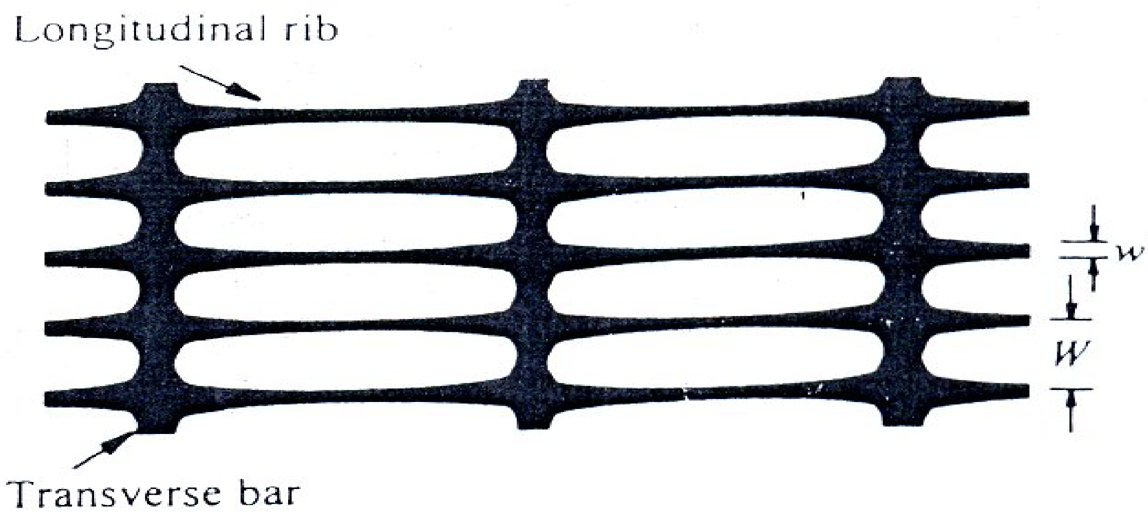
Fig. 2.1 Biaxial Geogrid



2. Uniaxial Geogrid

- Manufactured by stretching a punched sheet of extruded high density polyethylene in one direction.
- Has high tensile strength and modulus in one direction.

Fig. 2.2 Uniaxial Geogrid



2.2.2 Dimensions of Geogrids

- The grid apertures are either square, rectangular or elliptical
- Nominal rib thickness = 0.5 – 1.5 mm
- Junctions thickness = 2.5 – 5.0 mm
- Aperture dimension = 25 – 150 mm
- Open area of Geogrid > 50% of total area

2.2.3 Applications

Geogrids are typically used for:-

1. Slope Reinforcement: Highway embankments, overpasses, landslide or erosion-prone surfaces and landfill walls.
2. Base Reinforcement: Foundations for roads, parking lots, railroad track beds, airport tarmacs and runways.
3. Wall Reinforcement: Retaining walls, airport noise barriers, bridge supports and sea walls.
4. Berm Reinforcement: Spillway channels for earthen dams, levees and waste containment ponds.

Till date, several laboratory model tests have been carried out relating to the Load Bearing Capacity of Shallow Foundations supported by sand reinforced with various materials such as geogrids, geotextiles, rope fibers, metal strips and bars.

The use of geogrids for soil reinforcement has greatly increased over the last decade primarily because of following reasons:

1. Geogrids are dimensionally stable.
2. have high tensile modulus (that means low strain at high load)
3. open grid structure (that provides enhanced soil reinforcement interaction)
4. positive shear connection characteristics
5. light weight
6. long service life

2.3 MECHANISM OF LOAD BEARING AND FAILURE

2.3.1 Unreinforced case

The bearing capacity of foundation soil comes from cohesion factor, c and frictional factor, Φ. In granular soil (dry sand), load taking factor is only the frictional one. Safe bearing capacity is defined as the maximum pressure, which the soil can carry safely without the risk of shear failure. Shear failure may result from the foundation failure as well as from excessive settlement. Before the application of load, the soil below the base of the footing is in elastic equilibrium and after the load is applied, the soil passes from elastic to plastic equilibrium with failure.

The three principal modes of shear failure in soil are:

1. General Shear Failure: - The soil properties are assumed to be such that a slight downward movement of the footing develops fully plastic zones and the soil bulges out. It occurs in relatively incompressible soil. Dense sand having relative density greater than 70% fails under general shear failure.

2. Local Shear Failure: - Large deformations may occur below the footings before the failure zones are fully developed and is associated with considerable vertical soil movement before soil bulging takes place. This type of failure may take place in fairly soft or loose and compacted soil. Loose sand having relative density between 50-70% fails under local shear failure.

3. Punching Shear Failure: - No lateral movement takes place. When the load is increased, vertical movement of the footings occur and the soil surrounding the footing remains relatively in original position i.e. it does not take part in failure. Hence there will be no tilting of footing and no bulging of surface soil. This type of failure is expected in loose sand having greater compressibility and relative density less than 35%.

2.3.2 Reinforced case

The reinforced soil technique is based on the mobilization of the interfacial shearing resistance between the soil and reinforcement which in turn restrains the lateral deformation of the soil.

Soil below a footing consists of three zones, viz. an active Rankine zone (Zone 1) which moves vertically downwards and displaces the radial Prandtl zone (Zone 2) in a lateral direction and the passive Rankine zone (Zone 3) in upward direction.

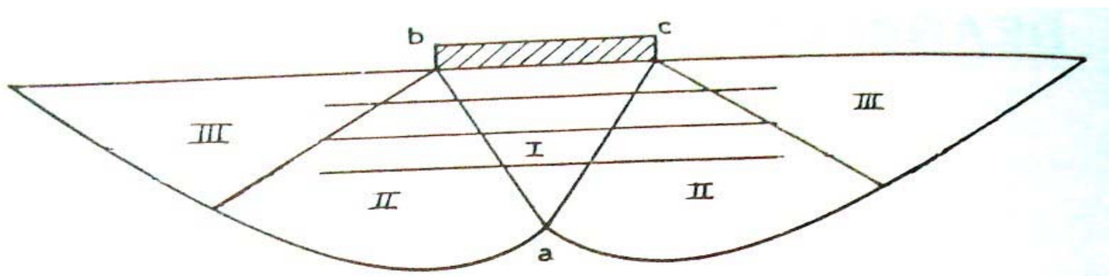


Fig 2.3 Interaction of reinforcements with failure wedge

In case of reinforced soil, the above possible failure surface is intercepted by the horizontally placed reinforcements. Therefore, for the lateral movement of zone 2 to occur, soil in that zone has to overcome the frictional resistance at the soil-reinforcement interface. Thus the effect of the reinforcement is to check the lateral flow of soil beneath the footing by introducing lateral confinement.

2.4 MECHANISM OF GEOGRID REINFORCEMENT

The provision of geogrid reinforcement imparts anisotropic mechanical properties, increased stiffness, tensile strengths, increased bearing capacity to the foundation soil.

Reinforcing mechanism comes from the interaction between geogrid and soil in one or more of the following way.

1. Soil shearing on plane surface of the grids (i.e. skin friction)
2. Soil bearing on lateral surface of grids (i.e. passive pressure resistance of the contact area between soils and geogrids)
3. Soil shearing over soil through the apertures of the grids (i.e. inter-facial shear on the surface of a rupture zone created during shearing)

The failure mechanisms are as follows:-

- Huang and Tatsuoka (1988, 1990) proposed "deep foundation mechanism" of failure. In this case, width of reinforcement, b = width of foundation, B and a quasi-rigid zone is developed beneath the foundation.

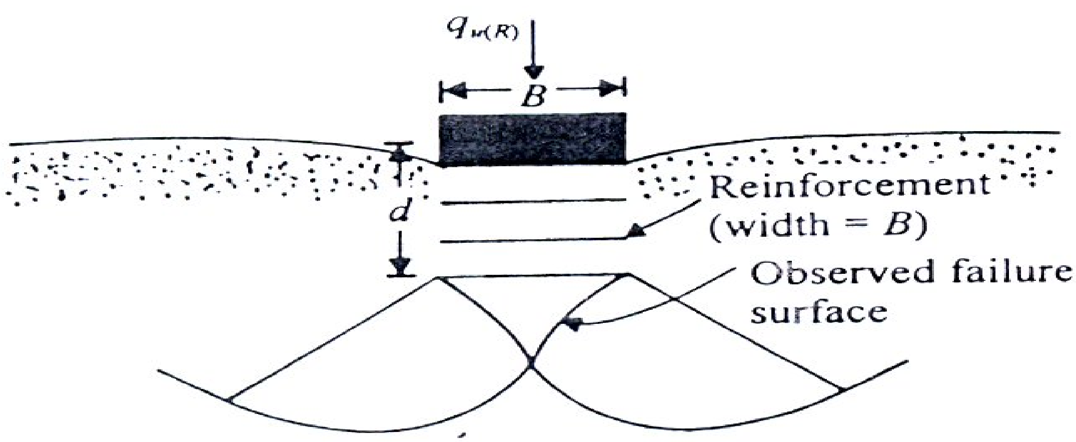


Fig 2.4 Deep footing failure mechanism in reinforced sand supporting a strip foundation (Huang and Tatsuoka, 1998)

- Schlosser (1983) proposed "wide slab mechanism" of failure in soil.
  In this case, b > B

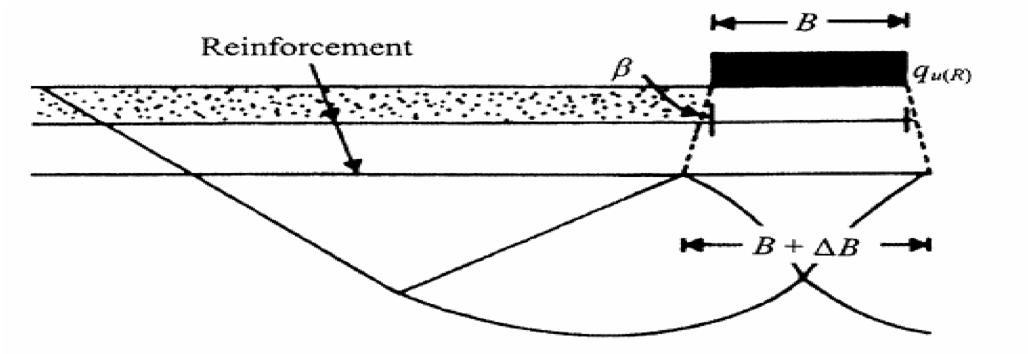


Fig 2.5 Wide-slab failure mechanism in reinforced sand supporting a strip foundation (Schlosser et al. 1983)

- Binquet and Lee (1975) proposed a rational design method. According, to this study, if layers of rein. are placed under a shallow strip foundation, the nature of failure in soil mass will be as follows :-

1. Shear failure of soil above the uppermost layer of the reinforcement. The mode of failure is possible if depth to the top most layer of reinforcement is sufficiently large, i.e. when $\frac{u}{B} \geq \frac{2}{3}$

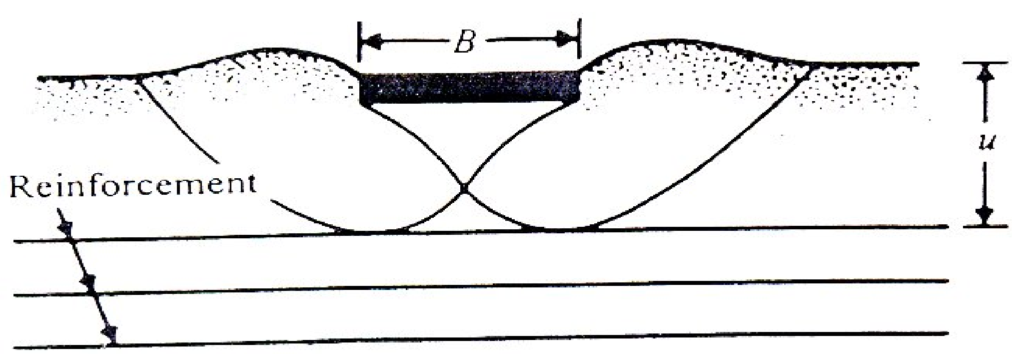


Fig 2.6 Bearing capacity failure above upper geogrid layer

2. Reinforcement pull out failure: This type of failure occurs for reinforcement placed at shallow depths beneath the footing with sufficient anchorage, i.e. when $\frac{u}{B} < \frac{2}{3}$, N = 2 – 3

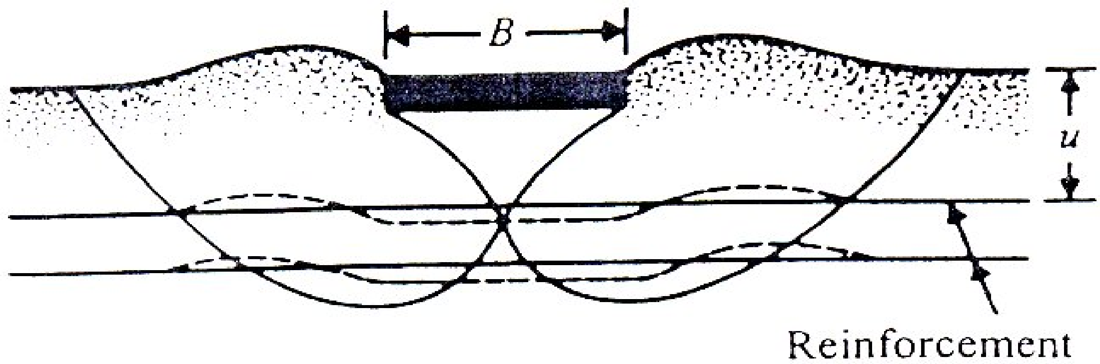


Fig 2.7 Anchorage pull out of geogrids due to deformation

3. Reinforcement tension failure: This type of failure occurs in the case of long and shallow reinforcement for which the frictional pull-out resistance is more than the tensile strength. The most beneficial effect of reinforced earth is obtained in this case, i.e. when $\frac{u}{B} < \frac{2}{3}$, and N > 4 but not more than 6 – 7.

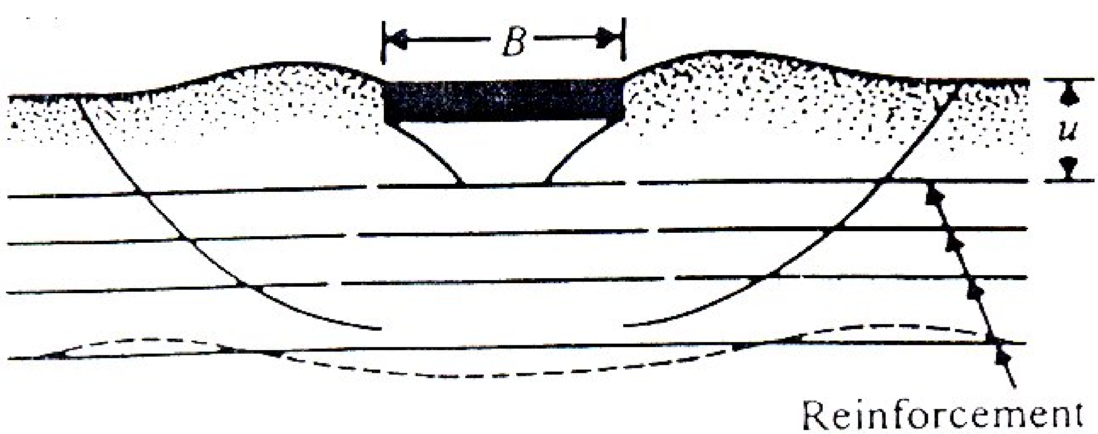


Fig 2.8 Tensile failure of geogrids

Chapter 3

EXPERIMENTAL SET UP AND PROCEDURE

3.1 SAMPLE COLLECTION

The sand collected from the river bed is made free from foreign matters i.e. roots, organic matters etc. and is cleaned by washing. Then it is oven dried and properly sieved, passing through 700 μ and retaining on 300 μ IS sieve. Dry sand is used as soil medium for the test as it does not include the effect of moisture and hence the apparent cohesion associated with it. Also due to non-availability of laboratory facilities, the conducting of test in a complex situation developed due to presence of moisture and cohesion has been avoided.

3.2 CHARACTERISTICS OF SAND

The characteristics of sand used are as follows:

1. Specific gravity G = 2.618
2. Maximum void ratio e_{max} = 0.995
3. Minimum void ratio e_{min} = 0.664
4. Relative density I_d = 72%
5. Dry density γ_d = 1.49 gm/cc
6. Angle of internal friction at the adopted density $\Phi = 42.34°$

Table 3.1 GRAIN SIZE ANALYSIS

Sl. No.	Sieve size (μ)	Wt. of sand retained (g)	% Retained	Cumulative % retained	Cumulative % finer
1	710	0	0	0	100.0
2	600	132.7	26.54	26.54	73.46
3	500	72.50	14.50	41.04	58.96
4	425	239.8	47.96	89.00	11.00
5	300	55.00	11.00	100.0	0

Fig 3.1 GRAIN SIZE DISTRIBUTION CURVE

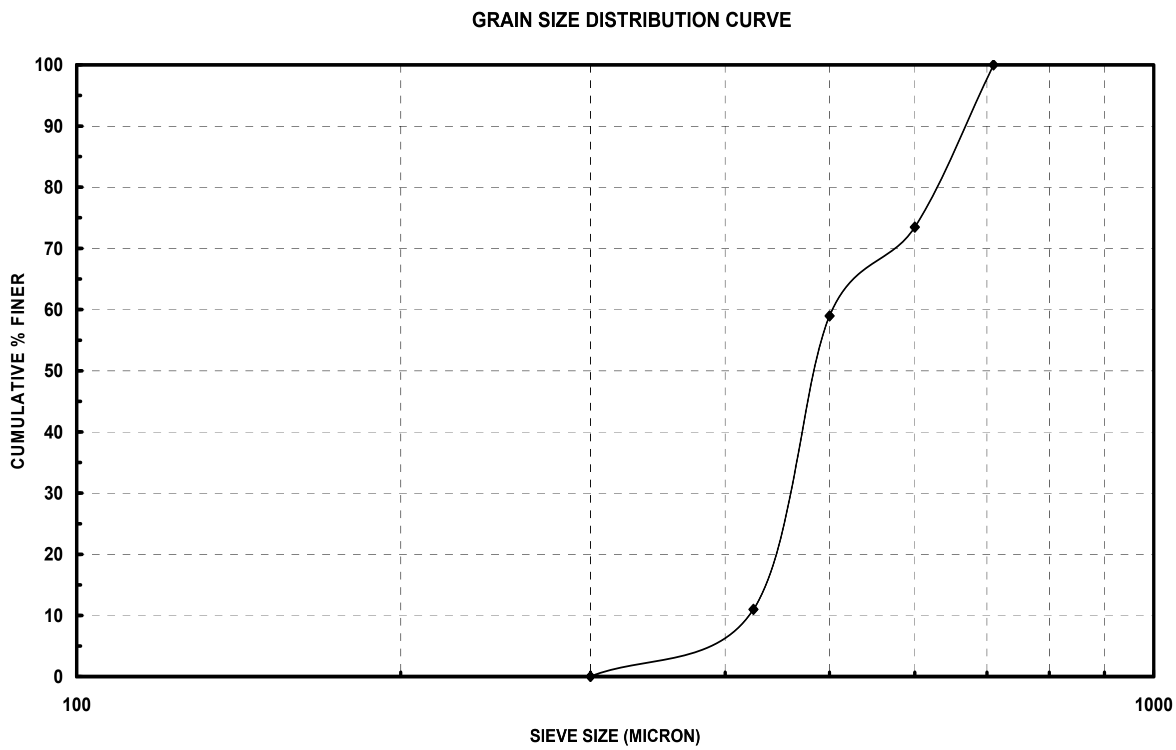
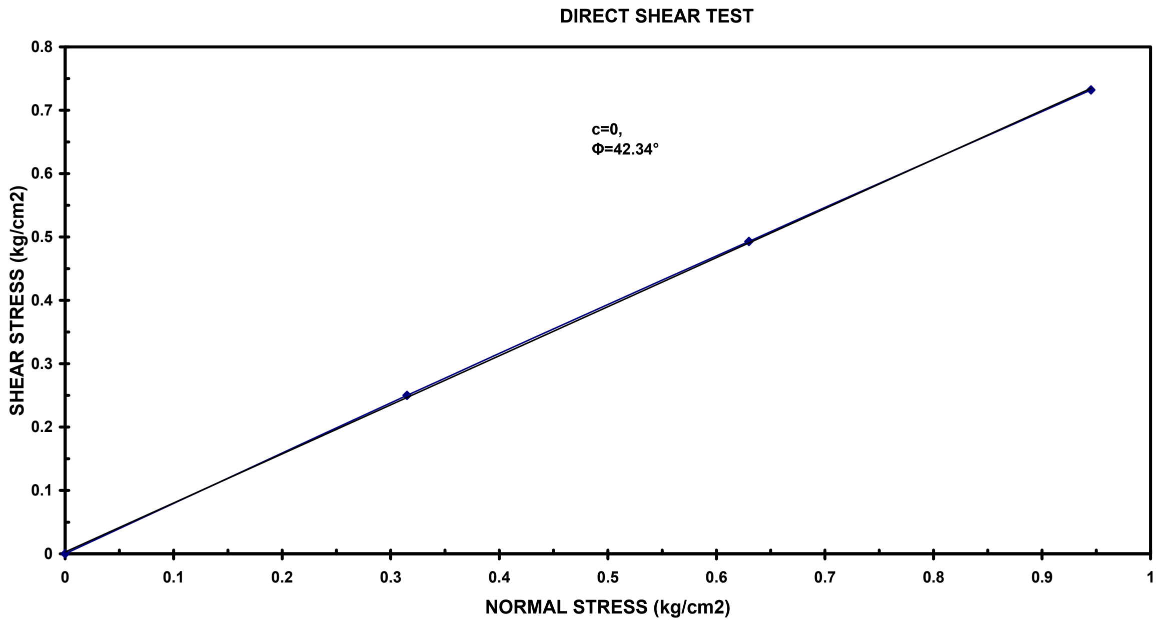


Table 3.2 DIRECT SHEAR TEST RESULTS

Sl. No.	Normal stress (kg/cm2)	Shear stress (kg/cm2)
1	0.315	0.309
2	0.630	0.523
3	0.945	0.811

Fig 3.2 PLOT OF SHEAR STRESS VS NORMAL STRESS



From the above graph; c = 0 and Φ = 42.34°

3.3 GEOGRIDS USED

Tensar BX1100 geogrid is used as reinforcement. The physical and mechanical properties of the geogrid as listed by the manufacturer are given below:

Polymer	polypropylene
Aperture shape	rectangle
Aperture size (MD/XD) (mm)	25/33
Rib thickness (mm)	0.75
Junction thickness (mm)	2.80
Tensile strength at 5% strain (KN/m) XD	8.46
Tensile strength at 5% strain (KN/m) MD	13.42
Long term allowable strength in crushed aggregated MD	N/A

(N.B. MD – Machine direction, XD – Cross machine direction)

3.4 TEST TANK

Bearing capacity tests were conducted in a box measuring 100 cm x 37 cm x 65 cm and made up of mild steel of 8 mm thickness. Scales are fitted on the internal walls of the box so that it will be helpful in maintaining the required density accurately. The sides of the box are heavily braced to avoid lateral yielding. The following considerations are taken into account while deciding the dimension of the box.

- As per provision of IS 1888-1962, the width of the test pit should not be less than 5 times the width of the test plate, so that the failure zones are freely developed without any interference from sides.
- Chumar (1972) has suggested that in case of cohesion less soil, the maximum extension of failure zone is 5B to the sides and 3B below the footing.

By adopting the above box size for the model footing (8 cm x 36 cm), it is ensured that the failure zones are fully and freely developed without any interference from the sides and bottom of the tank.

3.5 EQUIPMENTS USED

1. Static hydraulic loading system
2. LVDT indicator
3. Load cell indicator
4. Model footing

3.5.1 Static hydraulic loading system

The Hydraulic Pressure Testing Equipment is designed to test concrete and soil samples at high pressure. The testing pressure can be set from zero to 115 bar to get pressing force of 10 T in Cylinder - 1 and 20 T in Cylinder - 2. The test piece is kept on the machine base. Test pieces up to 1000 mm x 1500 mm can be tested.

The equipment comprises of the following main units.

Hydraulic System
Test Stand
Electric Control Panel

HYDRAULIC SYSTEM

The system comprises of a 150 lit reservoir mounted on which the gear pump is placed along with the various hydraulic elements. The piston pump is driven by a 3.75 kW/ 5 HP, 1440 rpm AC motor.

The system maximum pressure is 115 bar. The system is provided with standard elements like pressure relief valve, return line filter, suction filter, pressure gauge with isolation valves, oil level and temperature indicator, etc.

The system is provided with the following hydraulic elements:-

GEAR PUMP

The gear pump develops the test pressure of 115 bar (max.). The pump gets the oil supply from the reservoir. The gear pump design pressure is 210 bar against required maximum system pressure of 115 bar.

ELECTRIC MOTOR

A 3.73 kW/5 HP, 1440 RPM, foot mounted AC motor drives the gear pump. The direction of rotation of the motor (and the pump) is marked on the motor body. The motor drives the pump through a geared coupling.

BREATHER

A breather cum oil filling cap is provided on the reservoir for filling of oil and maintain the inside pressure of the reservoir to atmosphere.

RETURN LINE FILTER

The return line filter filters the hydraulic oil returning to the reservoir from the system during the operation. A 20 μ filter is provided.

PRESSURE RELIEF VALVES

A proportionate pressure relief valve is used to control the pressure from zero to the maximum system pressure of 115 bar. The pressure is regulated by varying the coil voltage from 0 to 10 V DC through an electronic card.

The pressure required for both the cylinders are the same. Cylinder-1 gives 10 T and Cylinder-2 gives 20 T at the maximum operating pressure of 115 bar.

A ‘two position’ directional valve is used with the pressure relief valve to load/un-load the system pressure.

DIRECTIONAL VALVES

Five directional valves are provided, one for the movement of the ram of Cylinder-1, another for the movement of the ram of Cylinder-2, the third for positioning the Cylinder-1 (in the horizontal axis), the fourth for positioning of the Cylinder-2 (in the horizontal direction) and the fifth for taking the platen up and down.

TEST STAND

The Test Stand is of very rigid construction. The machine frame can be placed on concrete floor for operating. No foundation is required. The test piece is placed on the machine base. The base can accommodate 1500 mm x 1000 mm test piece. The platen has four positions which can be set as per the height of the test piece. The Platen is moved up and down with the help of two hydraulic cylinders actuated by two push buttons. For taking the platen up or down, the four positioning pins have to be removed and the two hydraulic cylinders are actuated by operating the push buttons. The 10 T and the 20 T pressing cylinders move on rollers. They are moved from left to right or right to left with the corresponding hydraulic cylinders. These positioning cylinders are actuated by push buttons.

ELECTRIC PANEL

The Hydraulic Pressure Testing Equipment is designed for semi-automatic operation. Loading/unloading of the test piece is however is manual. The main motor (hydraulic) is initially started with a push button. It can be stopped by another push button. A selector switch is provided for moving the platen up and down, taking the pressing cylinders left and right and for actuating the 10 T and the 20 T pressing cylinders.

In automatic cycle, the pressing time can be set as required. Also the pressing force can be programmed with respect to the pressing time. On completion of the pressing cycle the ram returns back to its 'home' position. The control voltage for the solenoid coils (for the hydraulic and pneumatic systems) is 220 V AC. Control voltage for the digital meters is 24 V DC.

3.5.2 LVDT indicator

Linearly Variable Displacement Transducer indicator is used to measure the settlement of the footing produced due to application of pressure on the footing. Its accuracy is up to 0.001 mm

3.5.3 Load cell indicator

This instrument is used to indicate the load applied on the footing due to increase in pressure after regular intervals of time. Accuracy is up to 1 kg for 10 T load cell indicator and 2 kg for 20 T load cell indicator.

3.5.4 Model footing

Model footing used for laboratory tests is made of mild steel plate of size 8 cm x 36 cm x 2.5 cm. The length of the footing is made almost equal to the width of the tank, in order to create plane strain conditions within the test arrangement. A cross-mark is made exactly at centre of the footing for the centric application of load on the footing.

3.6 SAMPLE PREPARATION

The internal dimensions of the box are measured accurately and volume for a required thick layer is calculated. After fixing the density at which all tests are to be done, the weight of the sand needed for a particular thickness of sand layer is calculated. The density is found out to be 1.49 gm/cc and the layer thickness is 2.5 cm. For maintaining the above density and layer thickness, required weight of sand is found out to be 13.78 kg. The box is filled by sand normally up to a depth of 30 cm and then rest 25 cm is filled by using sand raining technique. The height of fall to achieve the required density is determined by performing a series of trials with different height of fall. After filling of each layer, leveling was done using a plane wooden plate and a level.

First the test is done without reinforcement and for the test with reinforcement; the first geogrid layer is placed at a depth of 0.35B from the base of footing. The other subsequent layer of geogrid is placed at equal spacing of 0.25B. After putting the geogrids, small weights are placed on them to keep the geogrid in position and then the required weight of sand is poured over it using sand raining technique. As the thickness of geogrid is very small in comparison to the sand layer considered and grid has large openings, so it is taken that the required density to be maintained is not affected.

3.7 TEST PROCEDURE

1. Upon filling the tank up to the desired height, the fill surface is leveled and the footing is placed on a predetermined alignment such that the loads from the cylindrical ram will be transferred vertically to the footing.

2. Then the LVDT indicator is placed at a suitable position on the footing to measure the settlement of footing during the experiment. The LVDT digital indicator is set to 80. The load cell indicator is set to 280.

3. The static hydraulic loading system is switched on and the beam is moved up. The four pins are removed and then the beam is moved down to the suitable position and at this position the four pins are again inserted to keep the beam in locked position during experimentation.

4. The cylindrical ram is moved down to place it exactly over the cross-hair marked on the footing.

5. The NITAL software is started and a time limit is fixed to perform this experiment. The load is applied on the footing by increasing the pressure.

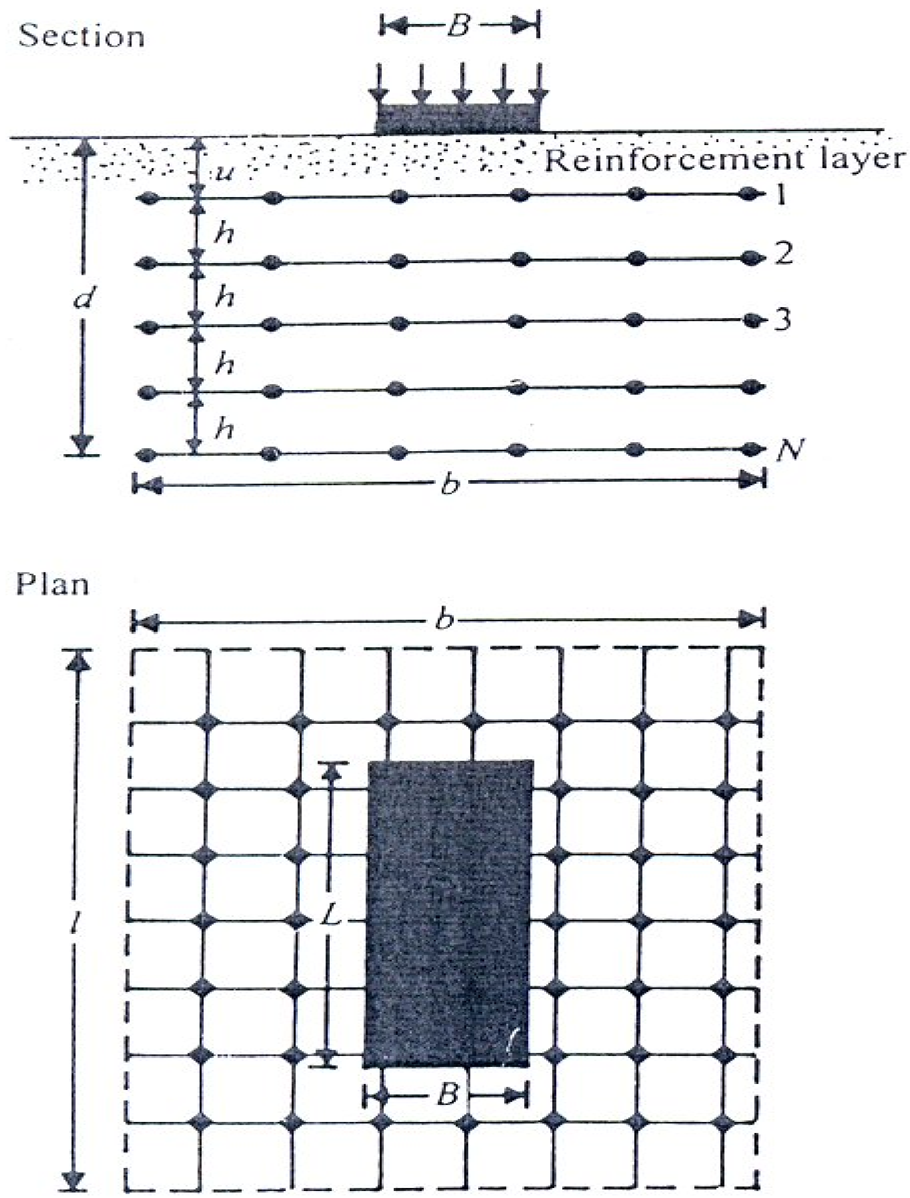
6. The load on the footing and the corresponding settlement are noted after regular intervals of time (say 5 min.).

7. The processes of load application is continued till there is failure of foundation sand due to sudden excessive settlement, which can be observed in the Load cell indicator where the load taken by the footing get decreased continuously.

8. On completion of the load test, he equipments are removed, box emptied and the box again refilled for the next set of load test.

3.8 GEOMETRIC PARAMETERS

This fig shows a strip foundation of width ‘B’ supported on Geogrid reinforced sand. There are four layers of geogrid, each having a width ‘b’. The top layer of geogrid is located at a depth u below the bottom of the foundation. The distance between consecutive layers of geogrid is ‘h’



Here, B = width of foundation

L = length of foundation

u = Dist of top layer of reinforcement from the bottom face of foundation.

d = Depth of foundation

b = Width of each reinforcement layer

l = Length of each reinforcement layer

$d = u + (N-1)\,h$

Where,

N – no. of reinforcement layers.

h – Vertical dist between 2 consecutive layers.

Hence, the total depth of reinforcement'd' bellow the bottom of the foundation is

$d = u + (4-1)\,h$

$= u + 3h$

In order to conduct model tests with geogrid reinforcement in sand, it is important to decide the magnitude of u/B, b/B and h/B to derive maximum benefit in increasing the ultimate bearing capacity. By conducting model tests on surface foundations supported by sand with multiple layer of reinforcement, it was shown by several previous investigators (Guido et al., 1987; Akinmusuru and akinbolande, 1981; yetimoglu et al., Shin and Das, 1990) that, for given values of h/B, u/B and b/B, the magnitude of BCR_u increases with u/B and attains a maximum value at $(u/B)_{cr}$. For $u/B > (u/B)_{cr}$, the magnitude of BCR_u decreases. By compiling several test results, Shin and Das (1999) determined that $(u/B)_{cr}$ for strip foundations can vary between 0.25 to 0.50. Keeping all these factors in mind, it is decided to adopt the following parameters for the present tests:

u/B = 0.35

h/B = 0.25

b/B = 4.50

Chapter 4

RESULTS AND DISCUSSIONS

From the test results, the load intensity versus the settlement curves are plotted. In each case, the ultimate bearing capacity is determined from the plotted graphs.

4.1 TESTS ON UNREINFORCED SAND

The test was conducted on unreinforced sand using 8cm width of footing. For vertical loading condition, the ultimate bearing capacity, Q_u of a strip foundation on unreinforced sand is expressed by following two established theories.

1. Terzhagi theory

   $q_u = 0.5\gamma BN_\gamma$ where N_γ = bearing capacity factor

   $N_\gamma = 2(N_q + 1) \tan\Phi$

   $N_q = e^{\pi\tan\Phi} \tan^2(\pi/4 + \Phi/2)$

   Using the above relationship, the theoretical ultimate bearing capacity for the present test conditions has been calculated.

   $N_\gamma = 185.44$

   $q_u = 1105.22$ gm/cm^2

   Ultimate load taken by the footing = 318.304 kg

2. Meyerhoff theory

   $q_u = 0.5\gamma BN_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma i}$

   $F_{\gamma s} = 1+ 0.1(B/L) \tan^2(\pi/4 + \Phi/2) = 1.114$

   $F_{\gamma d} = 1$

   $F_{\gamma i} = 1$

   Using the above relation it has been found out that –

   $q_u = 996.85$ gm/cm^2

   Ultimate load taken by the footing = 287.10 kg

The experimental data required for the determination of $q_{u\,(expt.)}$ are given as below:

Table 4.1 Load intensity Vs Settlement (N =0)

Sl. No.	Load Intensity (kg/cm^2)	Settlement (mm)
1	0	0
2	0.3	-0.27
3	0.5	-0.56
4	0.8	-0.90
5	0.973	-1.25
6	1.215	-1.65
7	1.469	-2.10
8	1.750	-2.75
9	2.058	-3.50
10	2.306	-4.30
11	2.554	-5.10

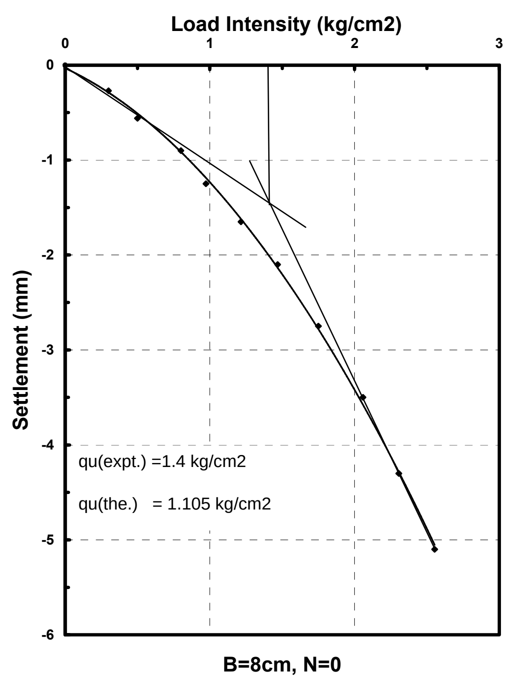


Fig 4.1 Determination of $q_{u\,(expt.)}$ for unreinforced case of loading

4.2 TESTS ON REINFORCED SAND

Tests were conducted on strip foundations supported on multilayered geogrid (BX1100) reinforcements at various depths of below base of foundation (i.e. d/B = 0.6, 0.85, 1.10) Huang and Menq (1997) have provided a tentative relationship to determine the ultimate bearing capacity of a strip foundation on reinforced sand based on wide slab mechanism.

The relationships can be expressed as:-

$q_{uR} = 0.5(B+\Delta B)\ \gamma N_\gamma + \ \gamma d N_q$

$\Delta B = 2d \tan\beta$

$\tan\beta = 0.68 - 2.071(h/B) + 0.743(CR) + 0.03(b/B) + 0.076N$

CR is Cover Ratio = w/W = 0.107

w = width of longitudinal ribs

W = centre to centre spacing of longitudinal ribs

$N_\gamma = 2(N_q + 1) \tan\Phi$

$N_q = e^{\pi \tan\Phi} \tan^2(\pi/4 + \Phi/2)$

$q_{uR} = 0.5(B+\Delta B)\ \gamma N_\gamma + \ \gamma d N_q$ is valid in the following ranges only.

$0 \leq \tan\beta \leq 1$

$0.25 \leq h/B \leq 0.5$

$0.02 \leq CR \leq 1.0$

$1 \leq b/B \leq 10$

$1 \leq N \leq 5$

$0.3 \leq d/B \leq 2.5$

Case 1:

$d/B = 0.6$, $N = 2$

$\tan\beta = 0.5289$

$\Delta B/B = 2(d/B)\tan\beta = 0.6347$

$q_{uR} = 2.253$ kg/cm^2

Case 2:

$d/B = 0.85$, $N = 3$

$\tan\beta = 0.6049$

$\Delta B/B = 1.028$

$q_{uR} = 2.908$ kg/cm^2

Case 3:

$d/B = 1.10$, $N = 4$

$\tan\beta = 0.6809$

$\Delta B/B = 1.498$

$q_{uR} = 3.639$ kg/cm^2

Case 4:

$d/B = 1.35$, $N = 5$

$\tan\beta = 0.7569$

$\Delta B/B = 2.0436$

$q_{uR} = 4.444$ kg/cm^2

The experimental values and the corresponding load intensity vs settlement graph have been obtained for the above mentioned conditions and are as follows:

Table 4.2 Load intensity Vs Settlement (N =2)

Sl. No.	**Load Intensity (kg/cm^2)**	**Settlement (mm)**
1	0	0
2	0.4	-0.28
3	0.8	-0.85
4	1.2	-1.52
5	1.6	-2.20
6	2.0	-3.10
7	2.4	-4.50
8	2.8	-6.10
9	3.2	-8.00
10	3.6	-9.50

Fig 4.2Determination of $q_{u\ (expt.)}$ for reinforced case of loading, N = 2

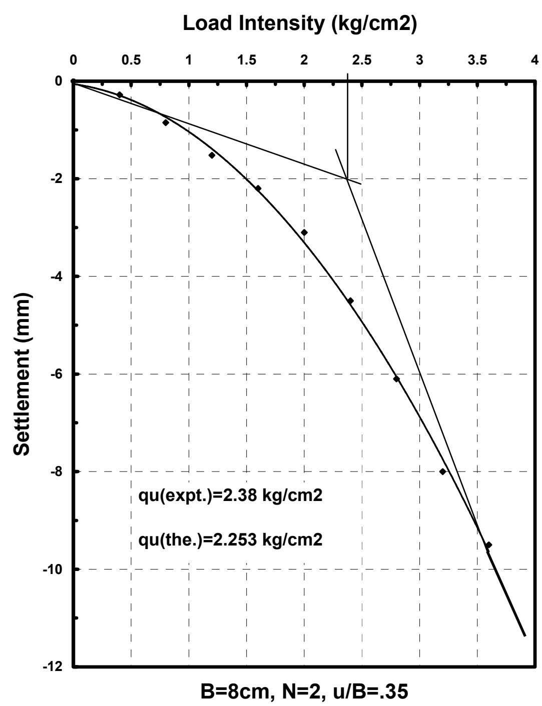


Table 4.3 Load intensity Vs Settlement (N =3)

Sl. No.	**Load Intensity (kg/cm^2)**	**Settlement (mm)**
1	0	0
2	0.4	-0.40
3	0.8	-0.72
4	1.2	-1.20
5	1.6	-1.70
6	2.0	-2.19
7	2.4	-2.90
8	2.8	-3.50
9	3.2	-4.20
10	3.6	-5.10
11	4.0	-5.80
12	4.4	-6.78
13	4.8	-7.70
14	5.2	-8.83
15	5.6	-9.80
16	6.0	-11.10
17	6.4	-12.40

Fig 4.3 Determination of $q_{u\,(expt.)}$ for reinforced case of loading, N = 3

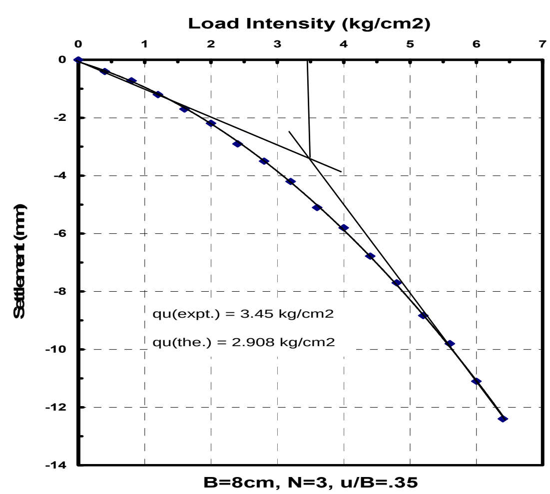


Table 4.4 Load intensity Vs Settlement (N =4)

Sl. No.	Load Intensity (kg/cm^2)	Settlement (mm)
1	0	0
2	0.8	-0.9
3	1.6	-1.6
4	2.4	-2.8
5	3.2	-3.9
6	4.0	-5.2
7	4.8	-6.3
8	5.6	-7.7
9	6.4	-9.2
10	7.2	-10.6
11	8.0	-12.3
12	8.8	-14.0
13	9.6	-15.8

Fig 4.4 Determination of $q_{u\ (expt.)}$ for reinforced case of loading, N = 4

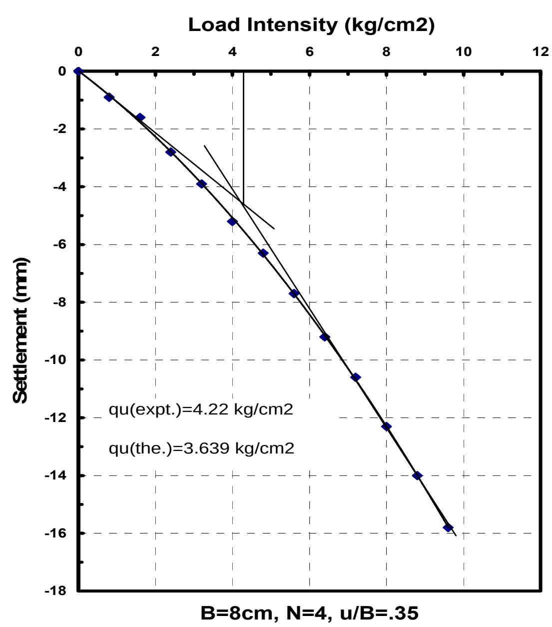
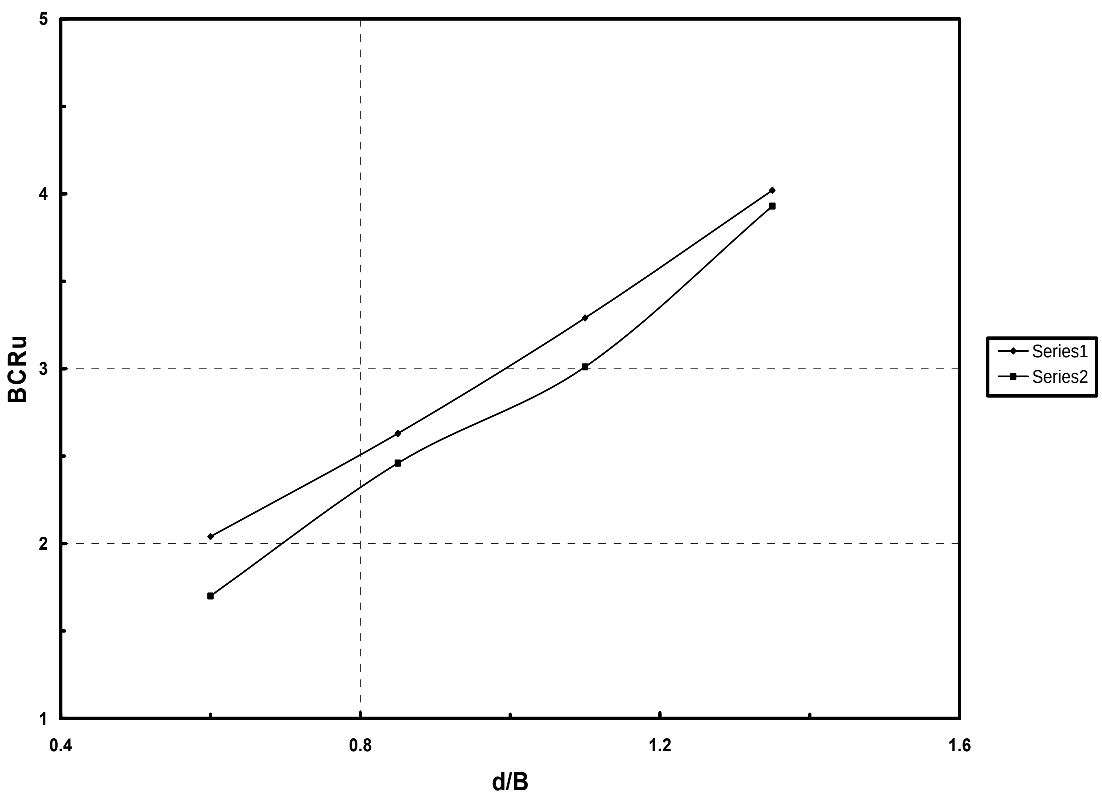


Table 4.5 Comparison between experimental and theoretical BCR_u

N	d/B	$q_{uR\ (the.)}$	$q_{uR\ (expt.)}$	$BCR_{u(the)}$	$BCR_{u(expt)}$
2	0.6	2.253	2.380	2.04	1.70
3	0.85	2.908	3.450	2.63	2.46
4	1.10	3.639	4.220	3.29	3.01

Fig 4.5 Variation of BCR_u with d/B



Series 1: theoretical value

Series 2: experimental value

Table 4.6 Comparison of experimental values of q_{uR} with values calculated from equation proposed by Huang and Menq, 1997.

N	$q_{uR\ (the.)}$ (kg/cm^2)	$q_{uR\ (expt.)}$ (kg/cm^2)	difference (kg/cm^2)	% difference (kg/cm^2)
2	2.253	2.380	0.127	5.64
3	2.908	3.450	0.442	15.20
4	3.639	4.220	0.681	18.71

Chapter 5

CONCLUSION

The ultimate bearing capacity obtained from the tests has been compared with the theoretical value developed by Huang & Menq, 1997. The following conclusions are drawn from the tests conducted in the present study:-

1. The experimental load carrying capacity of a foundation on homogeneous sand or on reinforced sand is always more than its theoretical load carrying capacity.

2. For the same soil, footing size and geogrid specification, the experimental and theoretical values of ultimate bearing capacity increase and the settlement increases with increase in number of geogrids.

3. The difference between experimental and theoretical values also increases with increase in number of geogrid layer. The maximum % difference between the experimental and theoretical values is 18.71%.

4. BCR_u increases with increase in d/B ratio. From fig 4.5, it is concluded that BCR_u would reach a maximum at some $d/B = (d/B)_{cr}$.

Chapter 6

SCOPE FOR FURTHER STUDIES

Keeping in view of the limitations on time, available laboratory facilities and its scope of present investigation, only a part of the problem was experimentally investigated. It is necessary to investigate the ultimate load at failure and the corresponding settlement in cohesive soil with geogrids as reinforcement.

Comprehensive investigation, both experimental and technical of the problem with geogrid as reinforcement is desirable.

Chapter 7

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